2. DEMANDS ON STRUCTURE COMPONENTS

2.1 Ground Motion Representation

Caltrans' Materials Engineering and Testing Service (METS) and Geotechnical Services (GS) will provide the following data defining the ground motion in the Preliminary Geology Recommendations (PGR).

- Soil Profile Type
- Peak rock acceleration for the Maximum Credible Earthquake (MCE)
- Moment magnitude for the MCE
- Acceleration Response Spectrum (ARS) curve recommendation
- Fault distance

Refer to Memo to Designers 1-35 for the procedure to request foundation data.

2.1.1 Spectral Acceleration

The horizontal mean spectral acceleration can be selected from an ARS curve. GEE will recommend a standard ARS curve, a modified standard ARS curve, or a site-specific ARS curve. Standard ARS curves for California are included in Appendix B. See Section 6.1.2 for information regarding modified ARS curves and site specific ARS curves.

2.1.2 Horizontal Ground Motion

Earthquake effects shall be determined from horizontal ground motion applied by either of the following methods:

Method 1 The application of the ground motion in two orthogonal directions along a set of global axes, where the longitudinal axis is typically represented by a chord connecting the two abutments, see Figure 2.1.

Case I: Combine the response resulting from 100% of the transverse loading with the corresponding response from 30% of the longitudinal loading.

Case II: Combine the response resulting from 100% of the longitudinal loading with the corresponding response from 30% of the transverse loading.

Method 2 The application of the ground motion along the principal axes of individual components. The ground motion must be applied at a sufficient number of angles to capture the maximum deformation of all critical components.

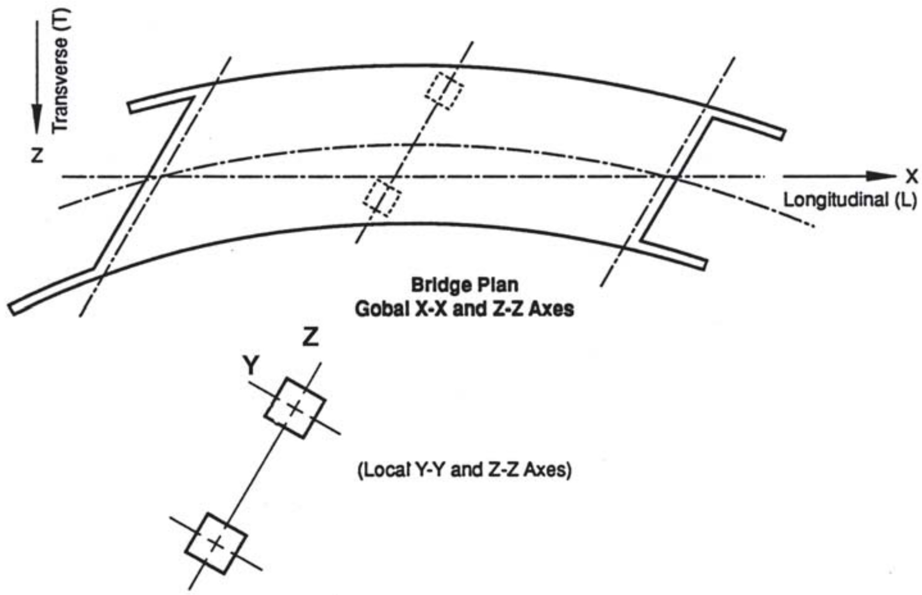


Figure 2.1 Local–Global Axis Definition

2.1.3 Vertical Ground Motion

For Ordinary Standard bridges where the site peak rock acceleration is 0.6g or greater, an equivalent static vertical load shall be applied to the superstructure to estimate the effects of vertical acceleration.[2] The superstructure shall be designed to resist the applied vertical force as specified in Section 7.2.2. A case-by-case determination on the effect of vertical load is required for Non-standard and Important bridges.

2.1.4 Vertical/Horizontal Load Combination

A combined vertical/horizontal load analysis is not required for Ordinary Standard bridges.

2.1.5 Damping

A 5% damped elastic ARS curve shall be used for determining the accelerations for Ordinary Standard concrete bridges. Damping ratios on the order of 10% can be justified for bridges that are heavily influenced by energy dissipation at the abutments and are expected to respond like single-degree-of-freedom systems. A reduction factor, R_D can be applied to the 5% damped ARS coefficient used to calculate the displacement demand.

[2] This is an interim method of approximating the effects of vertical acceleration on superstructure capacity. The intent is to ensure all superstructure types, especially lightly reinforced sections such as P/S box girders, have a nominal amount of mild reinforcement available to resist the combined effects of dead load, earthquake, and prestressing in the upward or downward direction. This is a subject of continued study.

The following characteristics are typically good indicators that higher damping may be anticipated [3].

- Total length less than 300 feet (90 m)
- Three spans or less
- Abutments designed for sustained soil mobilization
- Normal or slight skew (less than 20 degrees)
- Continuous superstructure without hinges or expansion joints

$$R_D = \frac{1.5}{[40c+1]} + 0.5 \qquad (2.1)$$

$ARS' = (R_D)(ARS)$

c = damping ratio ($0.05 \leq c \leq 0.1$)

ARS = 5% damped ARS curve

ARS' = modified ARS curve

However, abutments that are designed to fuse (seat type abutment with backwalls), or respond in a flexible manner, may not develop enough sustained soil-structure interaction to rely on the higher damping ratio

2.2 Displacement Demand

2.2.1 Estimated Displacement

The global displacement demand estimate, Δ_D for Ordinary Standard bridges can be determined by linear elastic analysis utilizing effective section properties as defined in Section 5.6.

Equivalent Static Analysis (ESA), as defined in Section 5.2.1, can be used to determine Δ_D if a dynamic analysis will not add significantly more insight into behavior. ESA is best suited for bridges or individual frames with the following characteristics:

- Response primarily captured by the fundamental mode of vibration with uniform translation
- Simply defined lateral force distribution (e.g. balanced spans, approximately equal bent stiffness)
- Low skew

Elastic Dynamic Analysis (EDA) as defined in Section 5.2.2 shall be used to determine Δ_D for all other Ordinary Standard bridges.

The global displacement demand estimate shall include the effects of soil/foundation flexibility if they are significant.

2.2.2 Global Structure Displacement and Local Member Displacement

Global structure displacement, Δ_D is the total displacement at a particular location within the structure or subsystem. The global displacement will include components attributed to foundation flexibility, Δ_f (i.e. foundation rotation or translation), flexibility of capacity protected components such as bent caps Δ_b , and the flexibility attributed to elastic and inelastic response of ductile members Δ_y and Δ_p respectively. The analytical model for determining the displacement demands shall include as many of the structural characteristics and boundary conditions affecting the structure's global displacements as possible. The effects of these characteristics on the global displacement of the structural system are illustrated in Figures 2.2 & 2.3.

Local member displacements such as column displacements, Δ_{col} are defined as the portion of global displacement attributed to the elastic displacement Δ_y and plastic displacement Δ_p of an individual member from the point of maximum moment to the point of contra-flexure as shown in Figure 2.2.

2.2.3 Displacement Ductility Demand

Displacement ductility demand is a measure of the imposed post-elastic deformation on a member. Displacement ductility is mathematically defined by equation 2.2.

$$\mu_D = \Delta_D / \Delta_{Y(i)} \tag{2.2}$$

Where: Δ_D = The estimated global frame displacement demand defined in Section 2.2.2

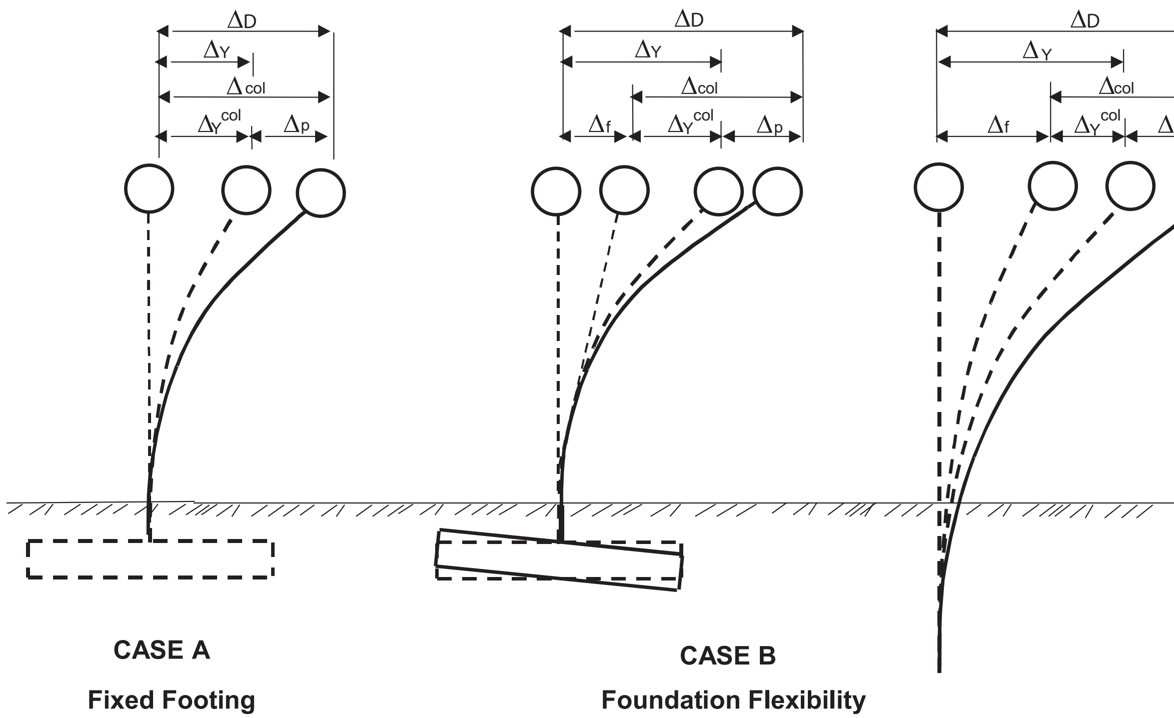
$\Delta_{Y(i)}$ = The yield displacement of the subsystem from its initial position to the formation of plastic hinge (i) See Figure 2.3

2.2.4 Target Displacement Ductility Demand

The target displacement ductility demand values for various components are identified below. These target values have been calibrated to laboratory test results of fix-based cantilever columns where the global displacement equals the column's displacement. The designer should recognize as the framing system becomes more complex and boundary conditions are included in the demand model, a greater percentage of the global displacement will be attributed to the flexibility of components other than the ductile members within the frame. These effects are further magnified when elastic displacements are used in the ductility definition specified in equation 2.2 and shown in Figure 2.3. For such systems, including but not limited to, Type I or Type II shafts, the global ductility demand values listed below may not be achieved. The target values may range between 1.5 and 3.5 where specific values cannot be defined.

Single Column Bents supported on fixed foundation	$\mu_D \le 4$
Multi-Column Bents supported on fixed or pinned footings	$\mu_D \le 5$
Pier Walls (weak direction) supported on fixed or pinned footings	$\mu_D \le 5$
Pier Walls (strong direction) supported on fixed or pinned footings	$\mu_D \le 1$

Minimum ductility values are not prescribed. The intent is to utilize the advantages of flexible systems, specifically to reduce the required strength of ductile members and minimize the demand imparted to adjacent capacity protected components. Columns or piers with flexible foundations will naturally have low displacement ductility demands because of the foundation's contribution to Δ_Y. The minimum lateral strength requirement in Section 3.5 or the P-Δ requirements in Section 4.2 may govern the design of frames where foundation flexibility lengthens the period of the structure into the range where the ARS demand is typically reduced.



Note: For a cantilever column w/fixed base $\Delta_Y^{col} = \Delta_Y$

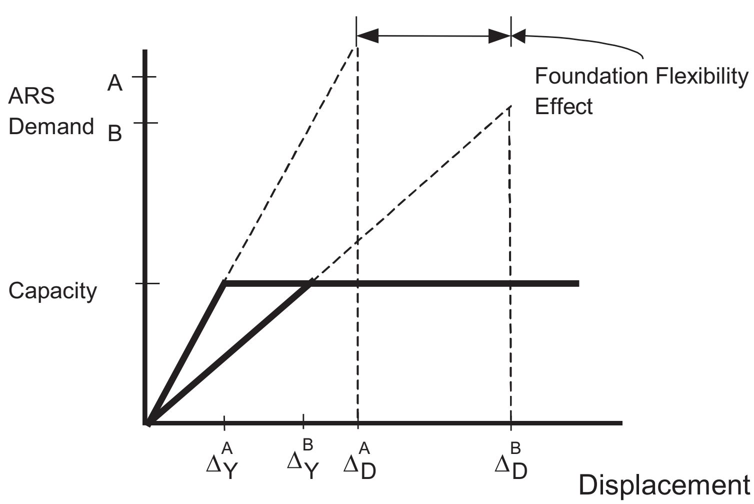
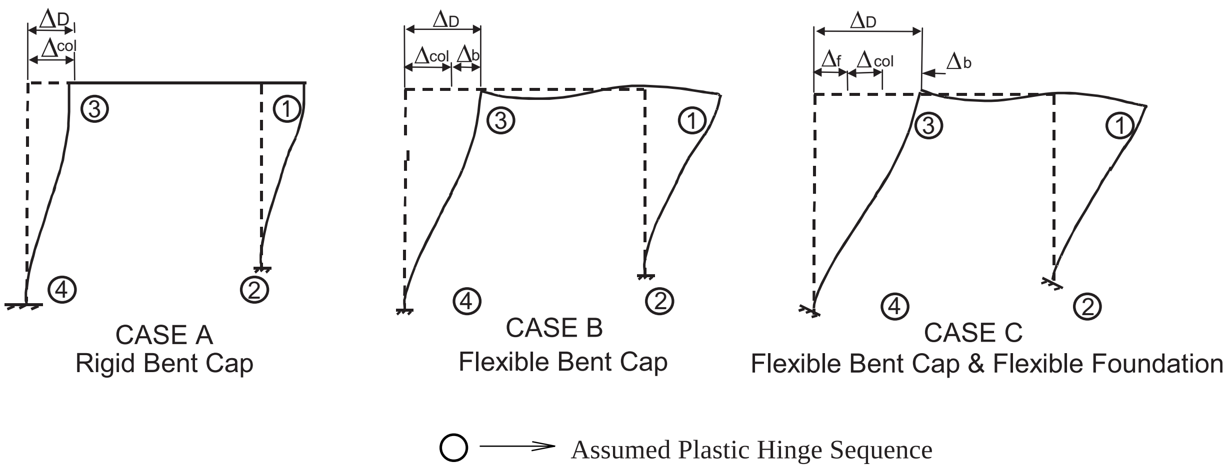


Figure 2.2 The Effects of Foundation Flexibility on Force-Deflection Curve of a Single Column Bent



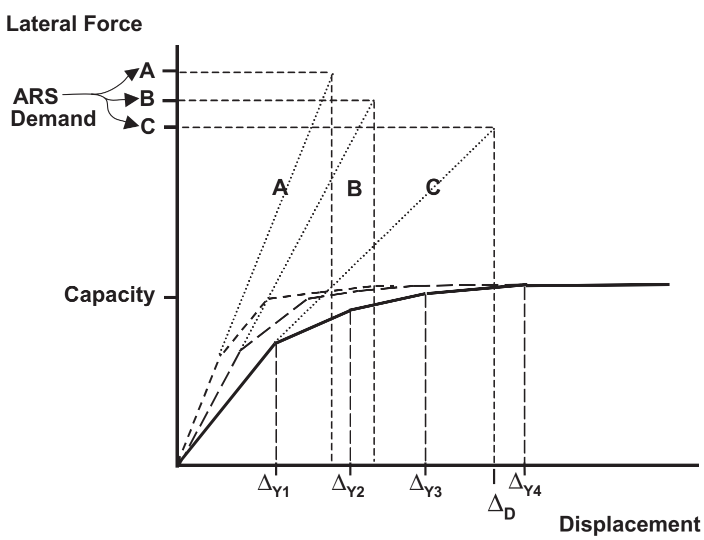
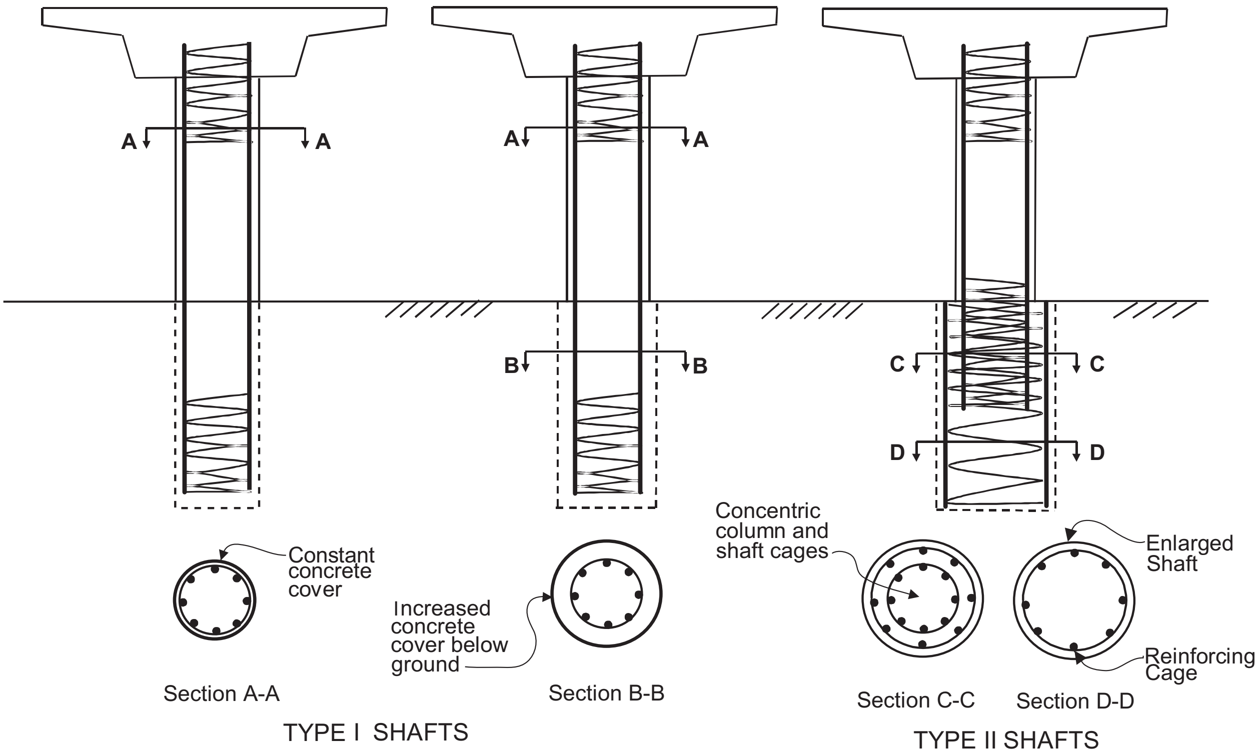


Figure 2.3 The Effects of Bent Cap and Foundation Flexibility on Force-Deflection Curve of a Bent Frame



Type I Pile Shafts

Type I pile shafts are designed so the plastic hinge will form below ground in the pile shaft. The concrete cover and area of transverse and longitudinal reinforcement may change between the column and Type I pile shaft, but the cross section of the confined core is the same for both the column and the pile shaft. The global displacement ductility demand, μ_D for a Type I pile shaft shall be less than or equal to the μ_D for the column supported by the shaft.

Type II Pile Shafts

Type II pile shafts are designed so the plastic hinge will form at or above the shaft/column interface, thereby, containing the majority of inelastic action to the ductile column element. Type II shafts are usually enlarged pile shafts characterized by a reinforcing cage in the shaft that has a diameter larger than the column it supports. Type II pile shafts shall be designed to remain elastic, $\mu_D \leq 1$. See Section 7.7.3.2 for design requirements for Type II pile shafts.

Figure 2.4 Pile Shaft Definitions

NOTE: Generally, the use of Type II Pile Shafts should be discussed and approved at the Type Selection Meeting. Type II Pile Shafts will increase the foundation costs, compared to Type I Pile Shafts, however there is an advantage of improved post-earthquake inspection and repair. Typically, Type I shaft is appropriate for short columns, while Type II shaft is used in conjunction with taller columns. The end result shall be a structure with an appropriate fundamental period, as discussed elsewhere.

2.3 Force Demand

The structure shall be designed to resist the internal forces generated when the structure reaches its Collapse Limit State. The Collapse Limit State is defined as the condition when a sufficient number of plastic hinges have formed within the structure to create a local or global collapse mechanism.

2.3.1 Moment Demand

The column design moments shall be determined by the idealized plastic capacity of the column's cross section, M_p^{col} defined in Section 3.3. The overstrength moment M_o^{col} defined in Section 4.3.1, the associated shear V_o^{col} defined in Section 2.3.2, and the moment distribution characteristics of the structural system shall determine the design moments for the capacity protected components adjacent to the column.

2.3.2 Shear Demand

2.3.2.1 Column Shear Demand

The column shear demand and the shear demand transferred to adjacent components shall be the shear force V_o^{col} associated with the overstrength column moment M_o^{col}. The designer shall consider all potential plastic hinge locations to insure the maximum possible shear demand has been determined.

2.3.2.2 Pier Wall Shear Demand

The shear demand for pier walls in the weak direction shall be calculated as described in Section 2.3.2.1. The shear demand for pier walls in the strong direction is dependent upon the boundary conditions of the pier wall. Pier walls with fixed-fixed end conditions shall be designed to resist the shear generated by the lesser of the unreduced elastic ARS demand or 130% of the ultimate shear capacity of the foundation (based on most probable geotechnical properties). Pier walls with fixed-pinned end conditions shall be designed for the least value of the unreduced elastic ARS demand or 130% of either the shear capacity of the pinned connection or the ultimate capacity of the foundation.

2.3.3 Shear Demand for Capacity Protected Members

The shear demand for essentially elastic capacity protected members shall be determined by the distribution of overstrength moments and associated shear when the frame or structure reaches its Collapse Limit State